

# Seismic intervention options for multi-tiered Nepalese Pagodas: The case study of Jaisedewal temple

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## ABSTRACT

During the 2015 Gorkha earthquake of 7.8  $M_w$  that hit Kathmandu Valley, Nepal, numerous Nepalese Pagodas suffered extensive damage while others collapsed. Risk reduction strategies implemented in the region focused on disassembling historical structures and rebuilding them with modern material without in depth analysis of why they suffer damage and collapse. The aim of this paper is to evaluate the effectiveness of low-cost, low-intervention, reversible repair and strengthening options for the Nepalese Pagodas. As a case study, the Jaisedewal Temple, typical example of the Nepalese architectural style, was investigated. A nonlinear three-dimensional finite element model of the Jaisedewal Temple was developed and the seismic performance of the temple was assessed by undertaking linear, nonlinear static and nonlinear dynamic analyses. Also, different structural intervention options, suggested by local engineers and architects working in the restoration of temples in Nepal, were examined for their efficacy to withstand strong earthquake vibrations. Additionally, the seismic response of the exposed foundation that the Nepalese Pagodas are sitting on was investigated. From the results analysis, it was found that pushover analysis failed to capture the type of failure which highlights the necessity to perform time-history analysis to accurately evaluate the seismic response of the investigated temple. Also, stiffening the connections along the temple was found to enhance the seismic behaviour of the temple, while strengthening the plinth base was concluded to be insignificant. Outputs from this research could contribute towards the strategic planning and conservation of multi-tiered temples across Nepal and reduce their risk to future earthquake damage without seriously affecting their beautiful architectural heritage.

## 1. Introduction

On April 25, 2015, the Gorkha earthquake of 7.8  $M_w$  struck close to the city of Kathmandu in Nepal [1–5]. It is estimated that during this earthquake, approximately 9000 people were killed, many thousands were injured, 2.8 million people were displaced, and more than half a million structures were either damaged or collapsed [6–9]. On top of that, dozens of Kathmandu heritage buildings suffered severe damages while others partially or totally collapsed [10,11]. In particular, numerous Nepalese Pagodas, exemplary temples of the Nepalese architecture, collapsed during the Gorkha earthquake [11–13]. The financial losses after the Gorkha earthquake have

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been estimated to be around 7 billion US dollars [14]. A side-effect of this earthquake was the severe drop in the influx numbers of tourists due to damages inflicted to the monuments of the Kathmandu region, entailing a significant drop in the income of the country. The significance of the conservation of the monuments of a country for the economic growth highlights the need for future-proofing actions in terms of restoration, rehabilitation and strengthening.

The excessive damage inflicted on the cultural heritage of Nepal urged researchers to investigate further the monuments that survived from the Gorkha earthquake and assist in the documentation of existing damages [15]. A rapid visual damage assessment survey was developed in the direct aftermath of the 2015 Gorkha earthquake to assess the safety and damage of residential buildings in the areas affected by the earthquake [16]. Digital recording and non-destructive techniques were utilized to assess the structural condition of historical temples [17], while a geo-crowdsourcing and web-mapping platform was developed incorporating both web and mobile based data collection with the intention to invite community participation [18,19]. This end-to-end system created an online virtual community encouraging public engagement and raising awareness about heritage ownership. It also provided valuable sources for cultural heritage exploitation, management, education, and monitoring over time. Furthermore, since masonry is one of the main structural components in modern and historical structures along the world, the research community has been focusing on developing methods for the efficient detection of damages in such constructions. Dais et al. [20] and Brackenbury et al. [21] developed algorithms for the automatic defect detection on photos of masonry structures. Researchers have introduced methods that automatically process point cloud representations of historic masonry structures and generate accurate numerical models that allow for efficiency and automation in structural assessment [22,23].

As discussed by Joshi & Kaushik [24], “loss of life” is more important than “loss of authenticity” and a more rational and practical stand on the future of Nepalese heritage is recommended to be adopted. In particular, different cases of restored or reconstructed historic structures that survived the Gorkha earthquake were presented. It was concluded that a degree of interference needs to be permitted to secure the Nepalese monuments against future earthquakes. Heritage structures, like the Nepalese Pagodas, are characterised by peculiarity and complex history. Thus, according to ICOMOS (International Council on Monuments and Sites), their assessment and retrofitting should require extra attention and involve the following steps: anamnesis, diagnosis, therapy and control [25]. In order to achieve cost effectiveness and minimal impact on architectural heritage using funds available in a rational way; it is usually necessary that the study repeats these steps in an iterative process. Regarding the anamnesis, the history of the Nepalese Pagodas highlights the high vulnerability of the temples. For instance, during the 1934 Nepal–Bihar earthquake ( $M_w$  8.0), over 200,000 buildings and temples were damaged [26]. This earthquake was well documented using photographs giving us clear evidence of the magnitude of devastation [24]. Several temples were reportedly heavily damaged during the 1934 earthquake and were reconstructed from the plinth level [27–30]. Consequently, interventions on the improvement of seismic performance of the Nepalese Pagodas are reckoned essential to reduce the seismic risk.

Masonry, a complex material consisting of units and mortar joints following a certain pattern, is heterogeneous and anisotropic in nature. Different approaches and scales exist in the literature for the numerical representation of masonry [31,32]. A detailed simulation of masonry, usually referred to as micro-modelling, includes the units, mortar and the unit/mortar interface [32,33]. This modelling approach enables an accurate representation of the complex response of masonry and is typically well suited for small structures. In larger structures such a modelling approach raises concerns regarding the increased computational cost and convergence issues. When assessing large scale structures, the masonry units and the mortar joints are commonly represented by homogenous elements, a method called macro-modelling [34]. A limitation of the macro-modelling approach is that it fails to reproduce the anisotropy in the inelastic range that characterises masonry [34]. The mortar joints act as planes of weakness [35] and thus the arrangement of bricks/blocks influences the orthotropic response of masonry [36,37]. Milani and Lourenço [38] in order to address this limitation examined representative elements of masonry walls with different orientations of the bed joints and the in-plane masonry strength was stochastically assessed. A continuum damage model was introduced by Pelà et al. [39] in which the orthotropic behaviour was simulated using the concept of mapped stress tensor. While such sophisticated numerical models exist in the literature, it is commonly accepted for damage-plasticity isotropic models mainly designed for concrete to be utilized to simulate masonry structures [37]. These isotropic models neglect any orthotropic effects but they have been found to reproduce accurately the overall behaviour of a structure [31,37].

Despite the considerable amount of work related to understanding the seismic vulnerability of historic temples in Nepal [11,28–30,40,41], little work has been done on the effectiveness of repair and strengthening options found in the literature (refer to Section 2.2). The aim of this paper is to evaluate the effectiveness of low-cost, low-interference and reversible repair and strengthening options for the Nepalese Pagodas. To this end, local engineers and architects working for the restoration of the temples in Nepal were engaged in different aspects of this study. The three-tiered Jaisedewal temple, a typical example of the Nepalese Pagoda architectural style, was chosen as a case study. A nonlinear three-dimensional finite element model of the Jaisedewal temple was developed in the commercial software ABAQUS based on the macro-modelling approach. Modal, pushover and time-history analyses were performed to assess the seismic vulnerability of the temple. Moreover, the plinth base, on which most of the Nepalese Pagodas sit, has attracted little research and thus its influence on the seismic response of the temple was investigated herein.

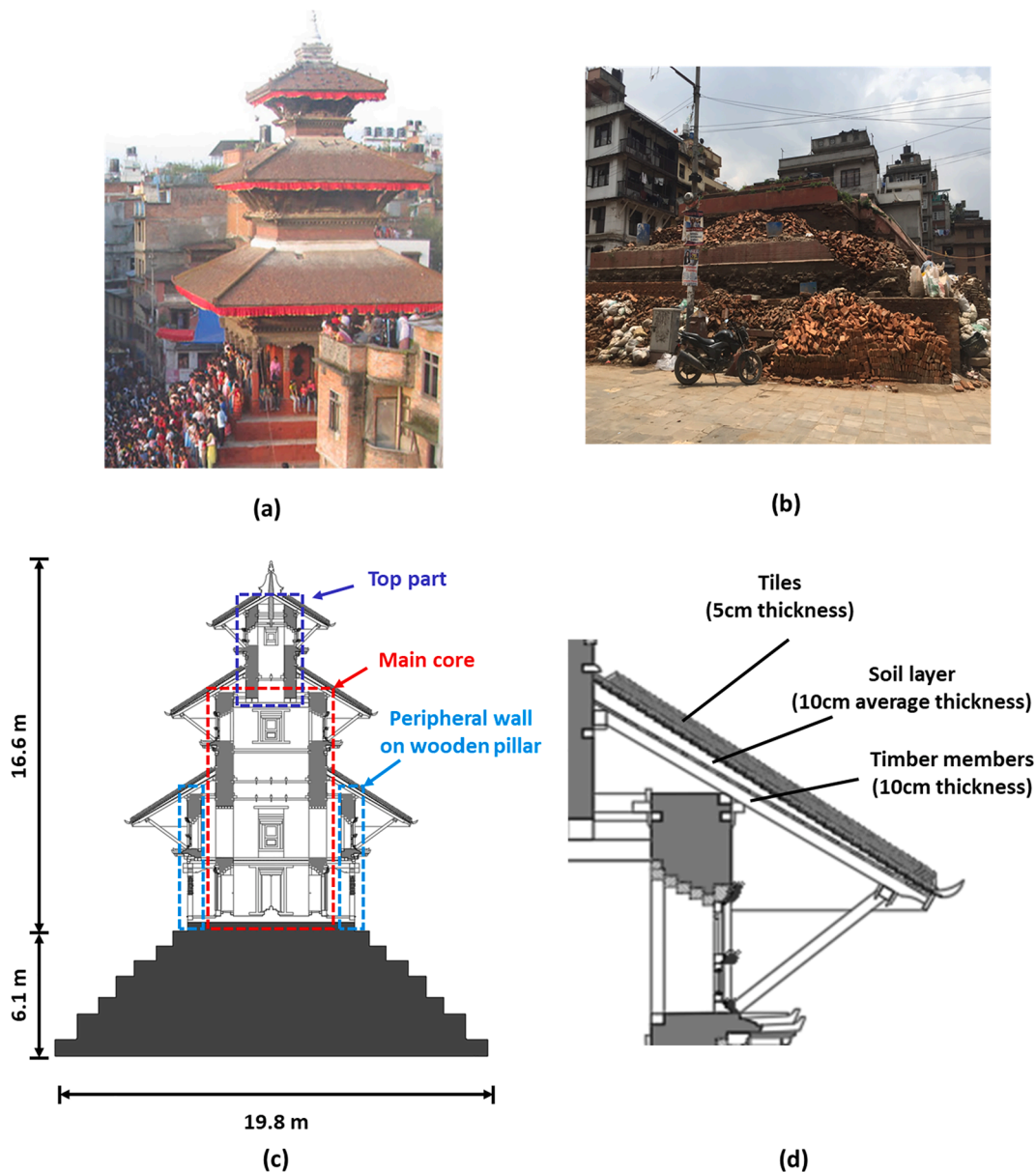
## 2. Case study and related work

### 2.1. Characteristics of the Nepalese Pagodas and the Jaisedewal temple

The Nepalese Pagodas, multi-tiered temples, began to appear extensively around the middle of the fourteenth century during the Malla Dynasty [28]. These monuments have been constructed using clay brick masonry and timber members with tiles or metal roof

coverings [42]. Most of the Nepalese Pagodas are considered non-engineered constructions that follow very simple rules and construction detailing with respect to the seismic resistance requirements [29]. Post-seismic surveys of past earthquakes, before the Gorkha earthquake, emphasized the high vulnerability of the Pagoda temples while excessive damages were expected in future earthquakes [29]. During the Gorkha earthquake, the pathologies of the Nepalese temples were revealed. The main causes of damage of these unreinforced masonry (URM) temples were: (a) deterioration of the construction materials; (b) lack of regular maintenance of the temples; (c) lack of proper connections in the corners; (d) excessive weight in the upper part of the building; and (e) discontinuous loading paths along the height of the structure [13,27,43,44].

The Jaisedewal temple (Fig. 1a) was chosen as case study and its seismic response was assessed. It is noted that the temple collapsed during the Gorkha earthquake and only the plinth base survived (Fig. 1b). The structure is a typical example of the Nepalese architecture in the Kathmandu Valley. The Jaisedewal temple is a three-tiered temple sitting on a massive exposed foundation plinth, the dimensions of which are  $19.8 \text{ m} \times 19.8 \text{ m}$  at the bottom and  $7.6 \text{ m} \times 7.6 \text{ m}$  at the top. The height of the plinth is  $6.1 \text{ m}$  while the height of the upper structure is  $16.6 \text{ m}$  (Fig. 1a). The exposed foundation has been constructed using red clay bricks bonded together by mud. The temple is characterized by a box-type configuration. Fig. 1c and d show a cross-section of the Jaisedewal temple as well as



**Fig. 1.** Jaisedewal temple: (a) before and (b) after the Gorkha earthquake, (c) cross section (illustration of foundation is indicative) and (d) detail of the roof system.

structural details of the roof system together with its different components (i.e. tiles, soil layer and timber elements). The walls of the temple were made of masonry clay units bonded together by brick mortar [8]. The structural system of the temple consists of a thick main core of walls in addition to a peripheral external wall sitting on wooden pillars (Fig. 1c). At the top of the temple there is an internal core with thinner cross-section, which does not sit on the main core of the structure but is simply connected with some crossing timber beams (Fig. 1c). For further details regarding the Jaisedewal temple, the reader is referred to the work of Kumar et al. [45] where a reconnaissance survey was undertaken and a geotechnical study on the site of the temple was presented.

Similar to the Jaisedewal temple, the Nepalese Pagodas are generally found on a massive plinth base (Fig. 1). The plinth is usually of low-quality workmanship and limited cohesion, thus of inferior performance than the supported temple [46]. According to [47], the Nepalese temples were placed on plinths in order to improve their seismic response against the wave amplification and to avoid the resonance with the soil layer. Although the structure could have been considered as an integral system consisting of the upper structure and the massive plinth base, the contribution of the latter has been neglected in previous numerical studies (see Section 2.2). In this study, the base plinth was incorporated in the numerical model and its influence on the overall response of the temple under seismic loads was investigated.

## 2.2. Numerical studies for the seismic assessment of Nepalese Pagodas

Regarding the numerical evaluation of the Nepalese Pagodas, in the literature, mostly linear elastic finite element modelling approach was utilized, while the plinth base was not incorporated in the model. In particular, modal analysis was performed and the dynamic characteristics of the structure were calibrated based on ambient vibration measurements [29,40]. Similar approach was followed by Shrestha et al. [27] in which static lateral load was applied and the maximum stresses were extracted for the modelled temple at its initial condition. A linear elastic finite element model was developed and calibrated over ambient vibration measurements, while different types of supports were investigated, including fixed and elastic support conditions [48]. A parametric study was undertaken on three different temples which were modelled adopting linear elastic material properties while neglecting the plinth base [28]. Degradation of different structural components was considered and simulated by decreasing the modulus of elasticity. The effect of this degradation on the fundamental frequency of temples was examined by Shakya et al. [28]. More elaborate numerical approach was followed and nonlinear material properties were considered by [49,50]. In particular, modal and pushover analyses were performed by Arce et al. [49], while time-history analyses were undertaken by Pejatovic et al. [50].

In terms of investigating the efficacy of retrofitting schemes little work could be found in the literature. A linear elastic finite element model was developed, static lateral load was applied and the maximum stresses were assessed to evaluate the effectiveness of a proposed retrofit which considered timber frames in both vertical and horizontal directions in the inner wall of the temple [27]. It is noted that such a retrofit scheme is highly intrusive and irreversible and thus its efficacy and necessity ought to be thoroughly examined before being implemented. Different strengthening methods which include the installation of timber and/or steel plates and the application of higher compressive strength of mud mortar masonry were investigated [49]. These solutions were investigated in terms of pushover analysis and the pseudo-static acceleration achieved was reported. It is noted that the examined interventions are highly invasive and milder approaches need to be considered first. Moreover, it was found that pushover analysis has many drawbacks and is not recommended when investigating complex structures such as the Nepalese Pagodas. The implementation of retrofitting solutions has been investigated in terms of assessment of its vulnerability and loss of structural integrity [43].

## 2.3. Common retrofitting practises for Nepalese Pagodas

Different interventions have been suggested for upgrading the seismic behaviour of the Nepalese Pagodas [10,24,43,46,51]. Some of the common practises are the stiffening of floors and strengthening of the traditional connections. In particular, timber floors with poorly connected elements and limited bracings, a common observation in the Nepalese temples, fail to provide diaphragmatic action and can be considered as rather flexible [48] (Fig. 2a). The replacement of the timber floors with stiffer elements can be part of the



Fig. 2. (a) Timber floors poorly connected with limited bracing elements [48]. (b) Rotting roof beams and planks [46].



maintenance of the Nepalese Pagodas given the fact that degradation has been observed in many cases [46] (Fig. 2b). Other typical examples of poor connection details are the timber beams that the top tier of the Pagodas usually sits on [49] (Fig. 3a) and the timber peg connections between the main masonry wall and the peripheral wall (Fig. 3b and c). The latter interventions are considered of low interference, since they are not visible and can be easily removed in order to bring the structure in its previous state.

Internal reinforcements and ties are applied to enhance the structural coherence of the building but have no visible impact on the outside. Further on, deep repointing and crack stitching has been suggested. Other common strengthening practises entail the installation of internal metal bracings or timber columns and beams in the existing structure. These interventions require new elements to be added to the temple and in most cases are irreversible.

Another intervention to reduce the seismic demand is the reduction of the mass of the structure. More specifically, the mass reduction can be achieved from the roofs and the masonry walls. Regarding the roofs, the soil layer (Fig. 1d) can be removed and the existing timber planks can be replaced with lighter ones. It is highlighted that the soil layer existing in the tiers could attract vegetation which diminishes the structural integrity of the timber roofs [10,46] (Fig. 4a) while rotten timber elements are typical observation during inspection of the Nepalese temples [51] (Fig. 4b). The restoration of the roofs is undertaken by local engineers and practitioners, while traditional techniques are followed and locally available hardwood, i.e. Shorea robusta, that is typically preferred for its strength and durability is installed adhering to the preservation of the architectural style of the Nepalese temples [51] (Fig. 4c and d). As far as the mass reduction from the masonry walls is considered, binding mortar is used and additionally hollow bricks could replace the existing solid ones which are heavier.

A common intervention applied on the Nepalese temples involves the removal of the existing bricks in the perimetry of the plinth base and replacing them with new masonry with higher strength characteristics (Fig. 5). Further information for the application of the intervention at the Silu Mahadev temple, which collapsed up to the plinth level during the Gorkha earthquake, are provided by GoN [52]. In detail, the outer shell of mud mortar plinth, two feet width is carefully removed and replaced by Lime Surkhi brick-wall without disturbing the inner core. This type of intervention has been practiced widely. Nevertheless, it affects the authentic style of the temples given that it is rather visible, while its efficacy has not been investigated yet.

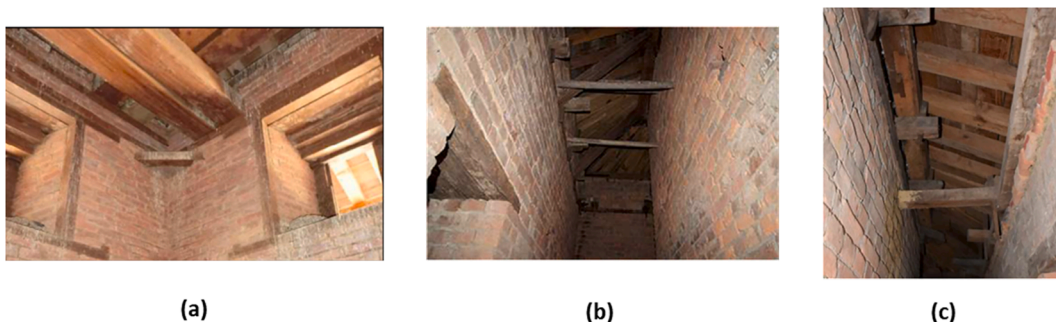
#### 2.4. Examined retrofitting solutions

In order to suggest a retrofit scheme on historical monuments the guidelines from ICOMOS ought to be taken into account [25]. Conservation, reinforcement and restoration of architectural heritage requires a multi-disciplinary approach. Any intervention on monumental structures is essential to be hidden or aesthetically pleasing, reversible and should comply with local material and construction methods. Moreover, no action should be undertaken without determining the likely benefit and harm to the architectural heritage. An incremental approach is suggested to be followed, beginning with a minimum level of intervention and the possible adoption of subsequent supplementary or corrective measures.

Taking all the above into consideration, local engineers and architects working for the restoration of the temples in Nepal were engaged in different aspects of this study in order to implement a multi-disciplinary assessment and select retrofitting solutions to be numerically studied. In particular, it was decided to evaluate the effectiveness of low-cost, low-interference and reversible repair and strengthening options. Moreover, following the guidelines of ICOMOS [25], an incremental approach was investigated. The stiffening of floors and strengthening of the traditional timber peg connections was examined together with the reduction of the mass of the temple. These interventions will be referred herein as “stiffened-floors” and “reduced-mass” respectively. As mentioned in Section 2.3, a common intervention on the Nepalese temples is to remove the existing bricks in the perimetry of the plinth base and replace them with new material. It was deemed important to examine the effectiveness of this intervention which is implemented even though it is irreversible, while its benefits have not been proven yet. This intervention will be hereafter called “new-stairs”.

### 3. Development of the computational model

A three-dimensional finite element model (Fig. 6) based on the macro-modelling approach has been developed in the commercial



**Fig. 3.** (a) The top tier of the Nepalese Pagodas usually sits on timber beams [49]. The traditional timber peg connections between the main masonry wall and the peripheral wall: (b) [48] and (c) [49].

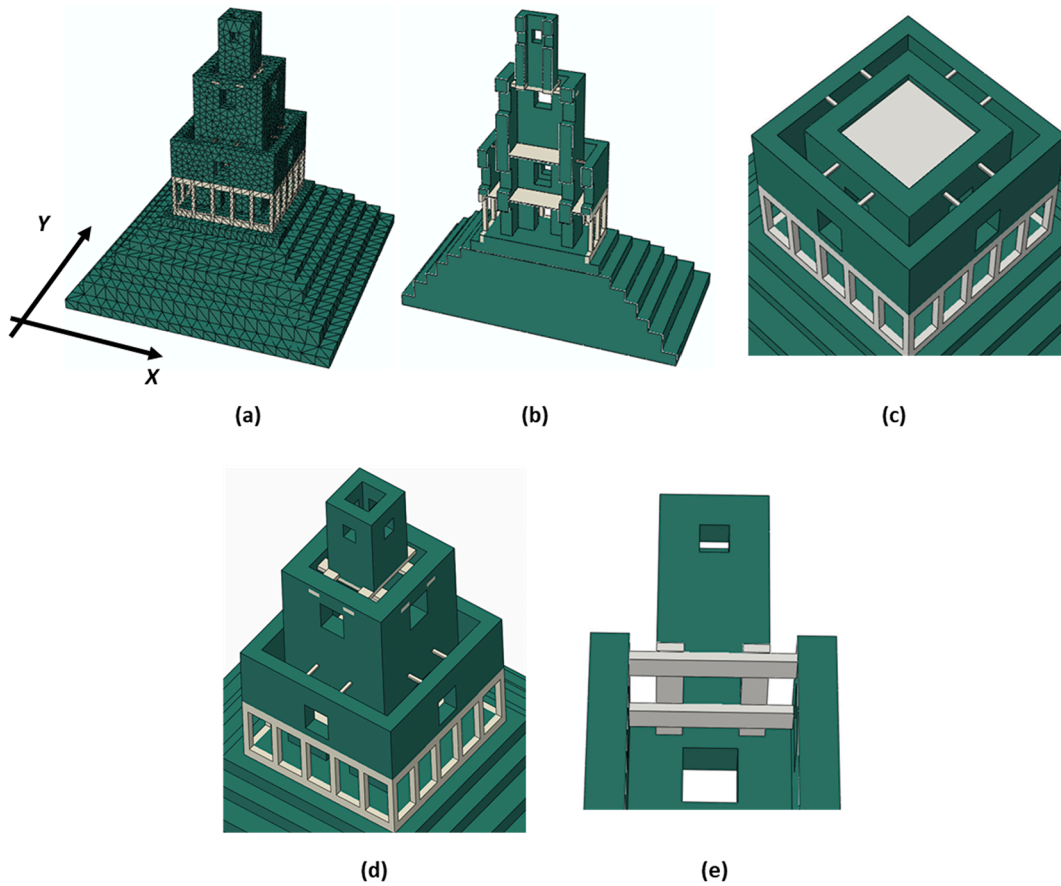


**Fig. 4.** (a) Vegetation attracted by the soil the soil layer existing in the tiers of the Nepalese temples [46]. (b) Rotten timber elements observed during inspections of the roofs of the Pagodas [51]. (c) and (d) Local material and techniques are used for the restoration of the roofs [51].

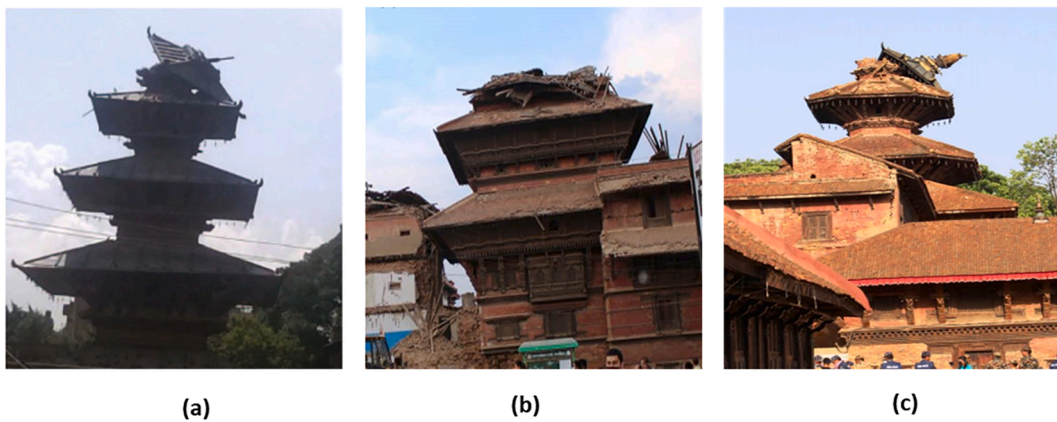


**Fig. 5.** A common intervention on the Nepalese temples is to remove the existing bricks in the perimetry of the plinth base and replace them with new material.

software ABAQUS [53] to analyse the seismic response of the Jaisedewal temple. The complex geometry of the temple was retrieved from drawings received by the Department of Archaeology in Nepal before collapse (Fig. 1) and was precisely incorporated in the numerical model. Due to the symmetry of the structure, loading was applied only along the horizontal (X-X') direction (Fig. 6). The structural members of the temple, i.e. timber parts and masonry walls, were simulated with 3D solid continuum elements (Fig. 6). A sensitivity study was considered in order to determine the size of the elements that yields the best compromise between accuracy and computational cost. In particular, it was found that with element size in the range of 450 mm the computational cost remained manageable in terms of analysis time while no reduction in the accuracy of the results was observed. The top masonry walls of the temples do not directly sit on the underlying masonry core and this could affect the performance of the temple to earthquake loads. An example of this connection detail is displayed in Fig. 3a, while the representation of the connection in the developed model is shown in Fig. 6e. This type of construction creates a discontinuous loading path. In fact, there were numerous examples of temples which survived the earthquake, but the top part collapsed (Fig. 7) [54,55]. Thus, special care was given to accurately reproduce this type of connection in the numerical model (Fig. 6). The timber elements were modelled as linear elastic while the masonry walls were modelled with nonlinear elements based on the Concrete Damage Plasticity (CDP) constitutive law [53]. This nonlinear constitutive law is suitable for the analysis of quasi-brittle materials, such as unreinforced masonry (URM), and to reproduce the main failure mechanisms of masonry i.e., cracking in tension and crushing in compression. This numerical approach has been used extensively to simulate historical URM structures [50,56–59]. The CDP constitutive law assumes different yield strength in tension and compression and stiffness degradation is incorporated in the post-yield phase. The degradation of the elastic stiffness is characterized by two damage variables,  $d_c$  and  $d_t$  for compression and tension respectively, and their evolution is a function of the plastic strains. If  $E_0$  is the initial (undamaged) elastic stiffness of the material, the stress-strain ( $\sigma - \epsilon$ ) relations under uniaxial tension and compression are:



**Fig. 6.** The developed initial model: (a) meshing, (b) vertical cross-section. (c) Horizontal cross section right above the top level of the peripheral wall. (d) and (e) display close-up details of the upper structure. Brickwork with mud mortar is shown with green colour and timber elements with grey colour. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)



**Fig. 7.** (a) Different cases of collapse of the top part of the Nepalese temples during the Gorkha earthquake that hit Nepal in 155, (a) [54], (b) and (c) [55].



$$\sigma_t = (1 - d_t)E_0(\varepsilon_t - \varepsilon_t^{pl}) \quad (1)$$

$$\sigma_c = (1 - d_c)E_0(\varepsilon_c - \varepsilon_c^{pl}) \quad (2)$$

where  $\varepsilon^{pl}$  is equivalent plastic strain; the subscripts ‘t’ and ‘c’ denote tension and compression respectively. The damage variables can take values from zero, representing the undamaged material, to one, which represents total loss of strength. In the developed model, the damage variables are up to 0.9. This is to allow for limited residual strength. Under uniaxial load the stress–strain response follows a linear elastic relationship until the value of the failure stress,  $\sigma_{t0}$  and  $\sigma_{c0}$  respectively for tension and compression is reached (Fig. 8). The failure stress corresponds to the onset of micro-cracking on the masonry walls. Beyond the failure stress, the formation of micro-cracks is represented macroscopically with a softening stress–strain response, which induces strain localization. The CDP model assumes non-associated plastic flow rule. The flow potential used for this model is the Drucker-Prager hyperbolic function [53]. The parameters to define the flow potential, yield surface and viscosity are given in Table 1. In particular,  $\sigma_{b0}/\sigma_{c0}$  is the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress; a default value equal to 1.16 was used. The parameter Kc denotes the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian for the yield function. A default value of the Kc equal to 0.666 was used (Fig. 9). This value distorts the Drucker-Prager surface rendering it closer to the Mohr-Coulomb criterion. The dilation angle was assumed to be equal to 10 degrees. This is in accordance with the experimental [60] and numerical studies undertaken on similar structures [31,56,59]. The default flow potential eccentricity ( $\varepsilon$ ) was taken equal to 0.1. This implies that the material has almost the same dilation angle over a wide range of confining stress values. Material models exhibiting softening behaviour and stiffness degradation often lead to severe convergence difficulties in implicit analysis programs. A common technique to overcome some of these convergence difficulties is the use of a visco-plastic regularization of the constitutive equations. Herein the viscosity parameter was 0.0005.

Due to the lack of experimental studies regarding the investigated temple a literature review on similar structures was performed to define the material properties for the numerical model. Several researchers collected detailed assessment data to characterize the material properties and the overall state of the temples in the aftermath of the Gorkha earthquake. Ambient vibration measurements were obtained on three Nepalese temples for the calibration of the corresponding numerical models concluding in an estimation of modulus of elasticity for the mud-mortar in the brick masonry multi-tiered temples in the range of 400–800 MPa [61]. Wood et al. [40] calibrated the numerical model of a five-tiered Nepalese Pagoda based on ambient vibration studies to represent the pre-earthquake condition and a value of 485 MPa was used for modulus of elasticity which falls close to the lower limit proposed by the work of Jaishi et al. [61], that is 400 MPa. Similarly, Shrestha et al. [27], while studying the seismic response of the Jagannath Temple, had to re-evaluate their initial estimation of the modulus of elasticity from 509 MPa and decreasing it by 80% in the upper wall due to cracks, and by 50% in the remaining walls in order to match the experimentally obtained fundamentals periods. UNESCO commissioned the development of a finite element model in order to get insight into the seismic vulnerability of the Gopinath temple in Hanuman Dhoka’s Durbar Square [49]. Though, initially a value of 800 MPa was used for the modulus of elasticity for the mud-mortar, the calibration of the model based on the ambient vibration measurements showed that a value of 150 MPa, well below the one originally used, was more realistic. Therefore, the modulus of elasticity of the Nepalese Pagodas could be classified in three levels: a) 150 MPa, b) 400 MPa and c) 800 MPa. In this study the first two levels were investigated and will be referred to as ‘low’ and ‘higher’ properties. Particularly, for the case of the brick walls, the modulus of elasticity was taken equal to 150 MPa and 400 MPa for the “low” and “higher” cases. Moreover, the value of compressive and tensile strength for the brick walls were chosen equal to 1 MPa and 0.05 MPa respectively [49]. The ability of the masonry walls to carry tensile forces is rather limited [62]. It is thus reasonable to assume the tensile strength as only 5% of the compressive strength. The modulus of elasticity for timber was 12,500 MPa [49]. The density of the brick walls and the timber elements were selected as 1800 kg/m<sup>3</sup> and 800 kg/m<sup>3</sup> respectively as proposed by [49,63]. The mass of the roof system (tiles, soil layer and timber members) was estimated in accordance with Nienhuys [46]. A mass of 1500 kg was assigned at the top of the structure to account for the decorative members at that level (Fig. 1a). The density of the tiles and the soil layer beneath was assumed equal to 1400 kg/m<sup>3</sup>. The material properties used for the development of the model are shown in Table 2. The masses of the three tiers were incorporated in the model as extra masses at the top of the masonry walls where the roofs were mounted to.

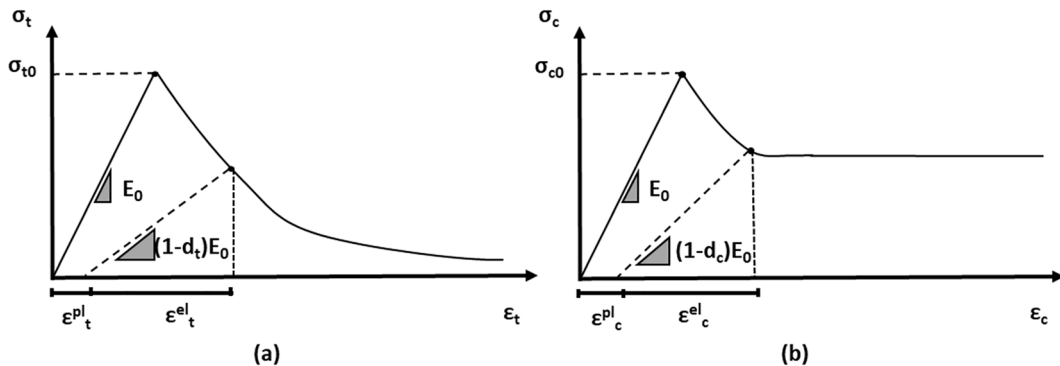
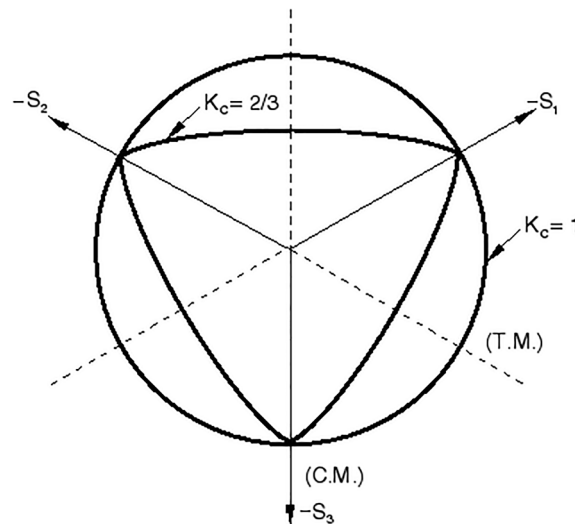


Fig. 8. The response of Concrete Damage Plasticity constitutive law to uniaxial loading in (a) tension, and (b) compression.



**Table 1**  
The main parameters for the Concrete Damage Plasticity model.

Dilatation angle [°]	Eccentricity	$\sigma_{b0}/\sigma_{c0}$	$K_c$	Viscosity
10	0.1	1.16	0.666	0.0005



**Fig. 9.** Yield surfaces in the deviatoric plane, corresponding to different values of  $K_c$  [53]. C.M.: compressive meridian and T.M.: tensile meridian.

**Table 2**  
Material properties used for the numerical models developed. E: modulus of elasticity,  $\nu$ : Poisson's ratio,  $f_c$ : compressive strength and  $f_t$ : tensile strength.

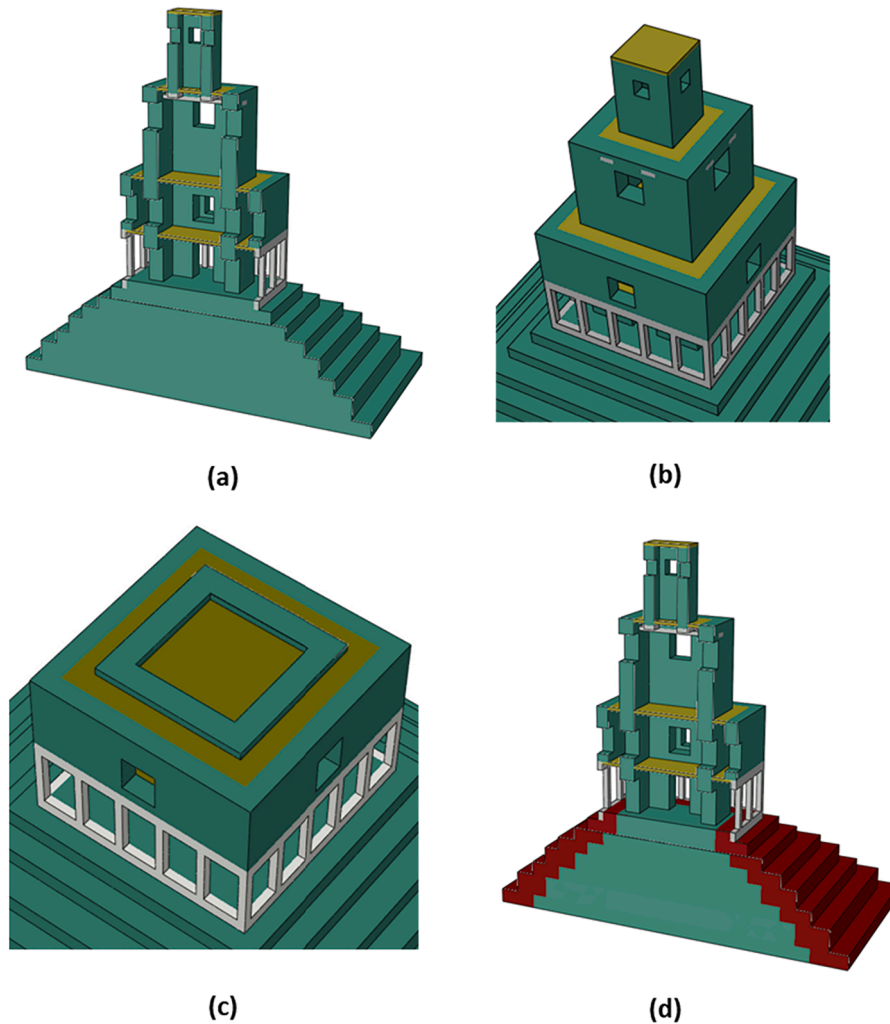
	E [MPa]	$\nu$	d [kg/m <sup>3</sup> ]	$f_c$ [MPa]	$f_t$ [MPa]	References
Mud-mortar brick masonry	150–400	0.24	1,800	1	0.05	[49,61,63]
Timber	12,500	0.3	800	–	–	[49]

The developed model will be referred to as “retrofitted model” or “initial model” whether any intervention has been applied to the upper structure of the temple or not, respectively. In particular, the initial model corresponds to the condition of the Jaisedewal temple prior to the Gorkha earthquake without any intervention on the upper structure. For this case, the wooden slabs were modelled as flexible providing limited diaphragmatic action to the structure (Fig. 6). The timber pegs that connect the peripheral walls with the main masonry core were also included in the numerical simulation (Fig. 6). As explained in Section 2.4, the interventions examined were namely stiffened-floors, reduced-mass and new-stairs. When the intervention stiffened-floors was applied, the floors were considered to be rigid enough to provide diaphragmatic action at the floor levels (Fig. 10a and b). Moreover, wooden planks were added to connect the main masonry core with the peripheral wall aiming to exclude the concentration of stresses around the connections of the existing timber pegs (Fig. 10a and b). As described in Section 2.3, the intervention of the new stairs is realized by replacing the existing material (that is mud mortar masonry) in the perimetry of the plinth base with new material (e.g. Lime Surkhi brick-wall [52]). This was numerically implemented by considering the modulus of elasticity of the stairs at the perimetry of the plinth base as 1000 MPa (Fig. 10d), which is higher than the values assigned to the rest of the plinth and the temple, which is 150 MPa or 400 MPa. The reduced-mass intervention was implemented by considering a 60% mass reduction of the roofs as per the instructions of Nienhuys [46].

Modal, pushover and time-history analyses were performed to assess the seismic response of the structure. The reference models for low material properties of the upper structure are the Initial\_E150\_2 and Retrofitted\_150\_2 and for higher properties are Initial\_E400\_2 and Retrofitted\_E400\_2 where 150 and 400 refer to the low and high modulus of elasticity respectively. Different modelling configurations of these reference models were developed in order to understand better the influence of the existence of the plinth base and the two further interventions (that is new-stairs and reduced-mass). All the different modelling configurations are reported in Table 3.

#### 4. Modal analysis

Firstly, modal analyses were performed for the initial and retrofitted models to investigate their dynamic characteristics. The influence of the material properties of the masonry walls (that is low or higher) and the simulation of the plinth base on the dynamic



**Fig. 10.** The developed retrofitted model with stiffened-floors: (a) cross-section and (b) detail of the upper structure. (c) Horizontal cross section right above the top level of the peripheral wall (d) The retrofitted model with the interventions stiffened-floors and new-stairs implemented. Brickwork with mud mortar is shown with green colour. Timber elements existing at the Jaisedewal temple before any intervention are shown with grey colour while timber elements added in the retrofitted model are displayed with yellow colour. Red colour denotes the new-stairs at the perimetry of the plinth base replaced with new material. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

characteristic of the temple were investigated.

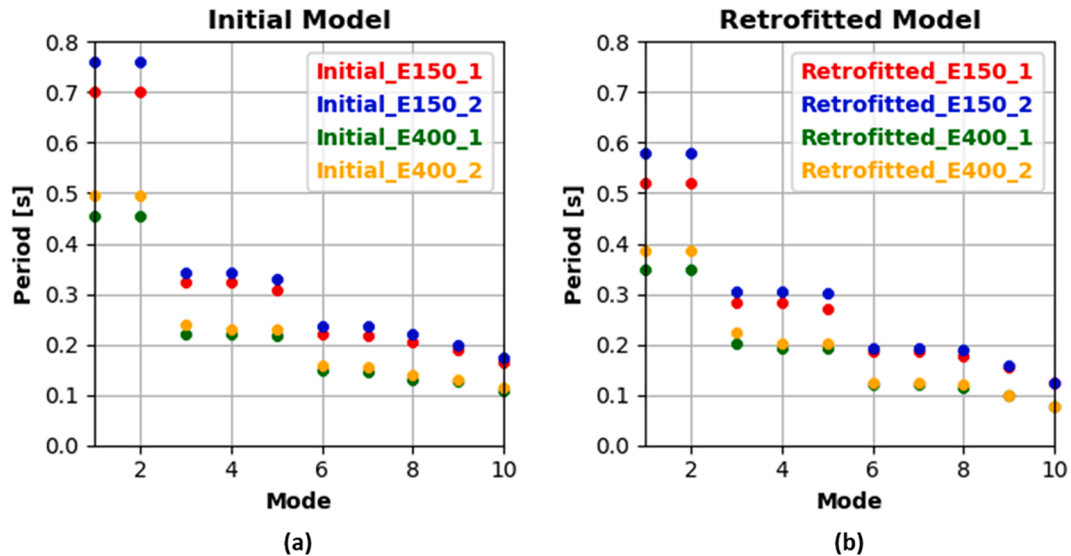
The periods of vibration of the first ten modes as calculated for different configurations of the initial and the retrofitted models are presented in Fig. 11. For all the different configurations examined, it was found that the first four modes were translational in the two horizontal directions of the temple, that is x and y (see Fig. 6a for a definition of the axes), while the fifth was torsional. The rest modes were local. It is noted that for the retrofitted models, the mode shapes did not alter significantly, but the obtained periods decreased up to 30%. Similarly, when the plinth base was ignored in the model, the first period of the temple reduced by approximately 10% but the type of the first ten modes did not change in comparison to the models containing the plinth base. Furthermore, the obtained periods, when higher material properties were considered, were profoundly reduced in comparison to the analyses with low material properties (see Fig. 11).

The first period of vibration for the initial (Initial\_E150\_2) and retrofitted (Retrofitted\_E150\_2) models, when low material properties were considered and the plinth base was modelled, were 0.76 s and 0.58 s respectively. The first mode of these two models is shown in Fig. 12. For the initial model Initial\_E150\_2, the peripheral walls tend to deform out-of-plane (OOP) due to the limited capacity of the wooden beams that connect the peripheral wall to the main masonry core. Consequently, this connection is flexible. The retrofitted model Retrofitted\_E150\_2 shows an enhanced behaviour when the first mode shape was examined (Fig. 12b). The peripheral wall moves along with the main core of the structure and the tendency for OOP deformations observed in the initial structure (Fig. 12a) has now been diminished.

**Table 3**

The different modelling configurations examined.

Reference Name	Plinth Base	Upper Structure Properties [MPa]	Plinth Base Properties [MPa]	Interventions	max Drift Ratio <sup>a</sup> [%]
Initial_E150_1	No	150	–	–	1.17
Initial_E150_2	Yes	150	150	–	1.30
Initial_E400_1	No	400	–	–	0.41
Initial_E400_2	Yes	400	150	–	0.56
Initial_E400_3	Yes	400	150	New-Stairs	0.44
Retrofitted_150_1	No	150	–	Stiffened-Floors	0.30
Retrofitted_150_2	Yes	150	150	Stiffened-Floors	0.47
Retrofitted_150_3	Yes	150	75	Stiffened-Floors	0.61
Retrofitted_150_4	Yes	150	150	Stiffened-Floors / New-Stairs	0.36
Retrofitted_150_5	Yes	150	150	Stiffened-Floors / Reduced-Mass	0.40
Retrofitted_E400_1	No	400	–	Stiffened-Floors	0.12
Retrofitted_E400_2	Yes	400	400	Stiffened-Floors	0.18
Retrofitted_E400_3	Yes	400	150	Stiffened-Floors	0.30
Retrofitted_E400_4	Yes	400	75	Stiffened-Floors	0.43
Retrofitted_E400_5	Yes	400	400	Stiffened-Floors / New-Stairs	0.17
Retrofitted_E400_6	Yes	400	400	Stiffened-Floors / Reduced-Mass	0.15

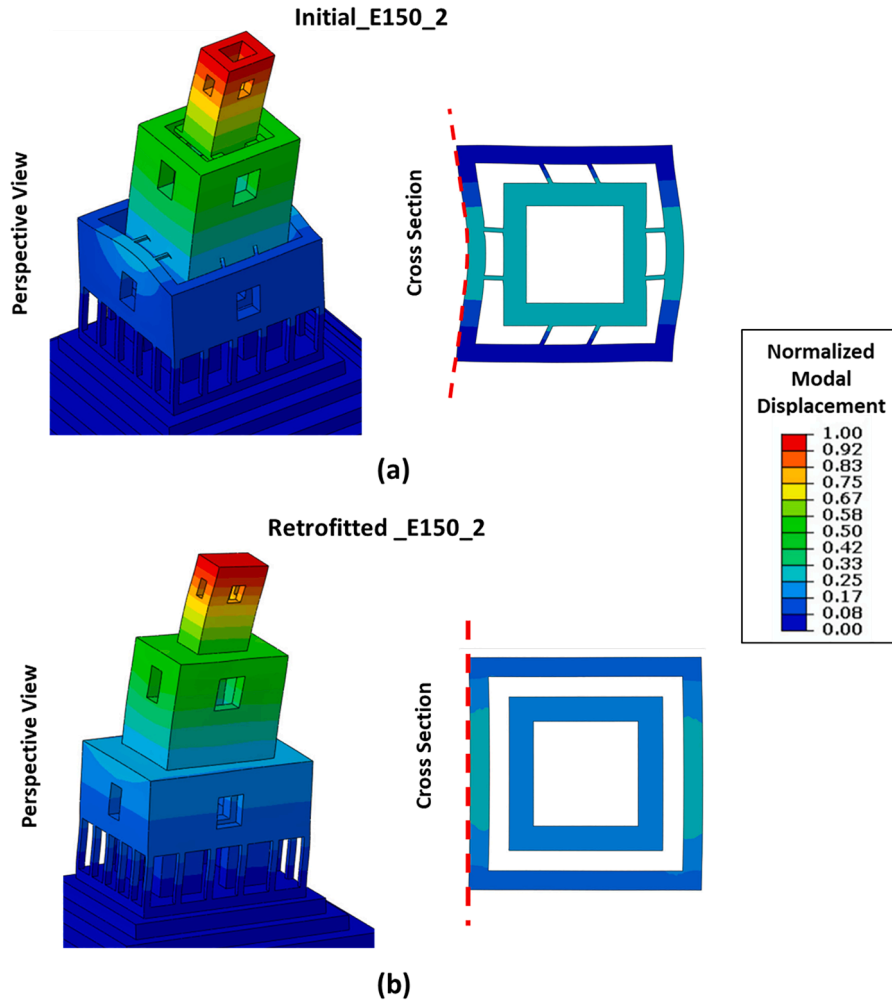
<sup>a</sup> Maximum drift ratio recorded at the top of the main core during the time-history analysis (see Section 6).**Fig. 11.** The first ten modes as calculated for different configurations of: (a) the initial and (b) the retrofitted models.

## 5. Pushover analysis

In order to better understand the nonlinear response of the case study to the earthquake loads, nonlinear static analysis with incrementally increasing horizontal loads was performed. The loads were applied proportionally to the mass of the structure attempting to imitate the horizontal forces of the seismic excitation. The horizontal load was increased until the structure attained its capacity in order to obtain the failure patterns of the temple. The pushover analysis was undertaken to evaluate whether it can provide reliable assessment of the Nepalese Pagodas which are tall structures with complex geometry. The findings from the pushover analysis were compared with the ones from the nonlinear time-history analysis in Section 6.

Fig. 13 shows the failure patterns for the initial (Initial\_E150\_2) and retrofitted (Retrofitted\_E150\_2) models, when low material properties are considered and the plinth base is modelled, as obtained from the pushover analysis. The results are presented for 0.5% drift ratio at the top of the main core, which is the value of drift ratio that corresponds to the yield limit for URM structures as defined in [64].

At the Initial\_E150\_2 model, concentration of damages occurs around the timber pegs which connect the main core to the peripheral wall (Fig. 13a). Due to the lack of diaphragmatic action at the level of the timber floors, the structure experiences extensive damage at the corners of the peripheral walls. Furthermore, the failure of the timber joist connections leads to OOP deformations in the middle of the peripheral wall, responsible for the development of cracks at these areas. Consequently, it can be inferred that the



**Fig. 12.** The first mode shape of the (a) initial (Initial\_E150\_2) and (b) retrofitted (Retrofitted\_E150\_2) models with mass participation ratios in the x direction (see Fig. 6a for the definition of x axis) 51% and 57% respectively. For each model a perspective view and a cross section at the top level of the peripheral wall are demonstrated in left and right respectively. The OOP motion of the peripheral wall is denoted with a red dashed line. The contours demonstrate the normalized modal displacement.

damages on the initial structure originate mostly from local effects. Similar failure patterns were observed in all the investigated modelling configurations of the initial model.

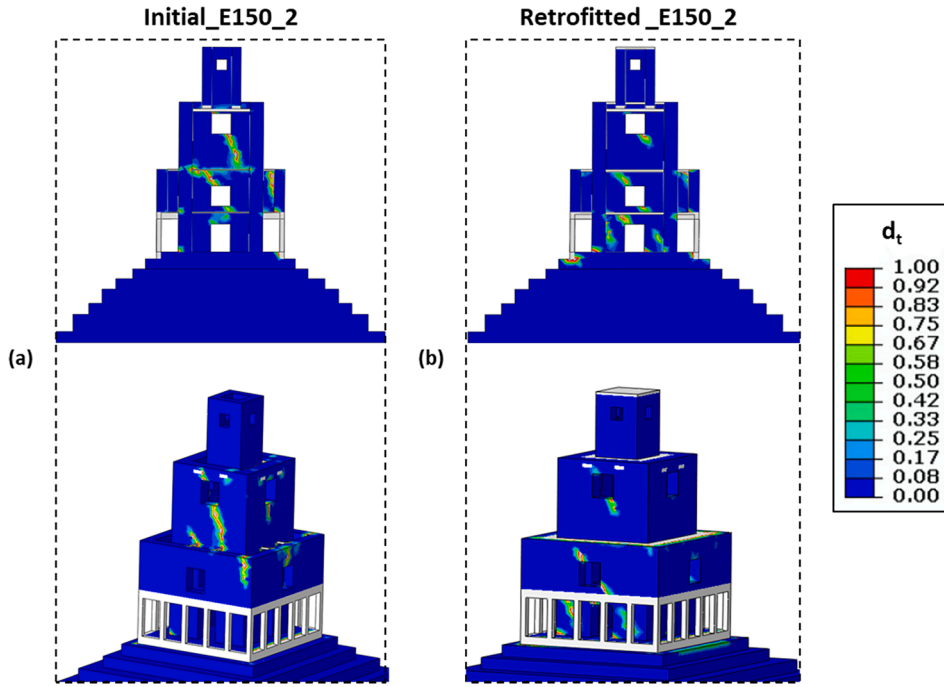
Fig. 13b shows the failure patterns for the retrofitted model Retrofitted\_E150\_2 as obtained from the pushover analysis for 0.5% drift ratio at the top of the main core. The main damages manifest as diagonal cracks on the main core. The tendency of the peripheral wall for OOP deformations has been eliminated and the cracks at the corners of the peripheral walls are limited. A limitation of the retrofitted structure is that the wooden pillars participate more in carrying horizontal loads, thus higher stresses are transmitted to their foundation causing the formation of cracks. Consequently, the foundation of the wooden pillars needs to be checked if it can safely carry these extra horizontal loads. After the retrofit has taken place, the whole temple follows the deformed shape of the main masonry core and the impact of local failure modes is limited.

## 6. Nonlinear time-history analysis

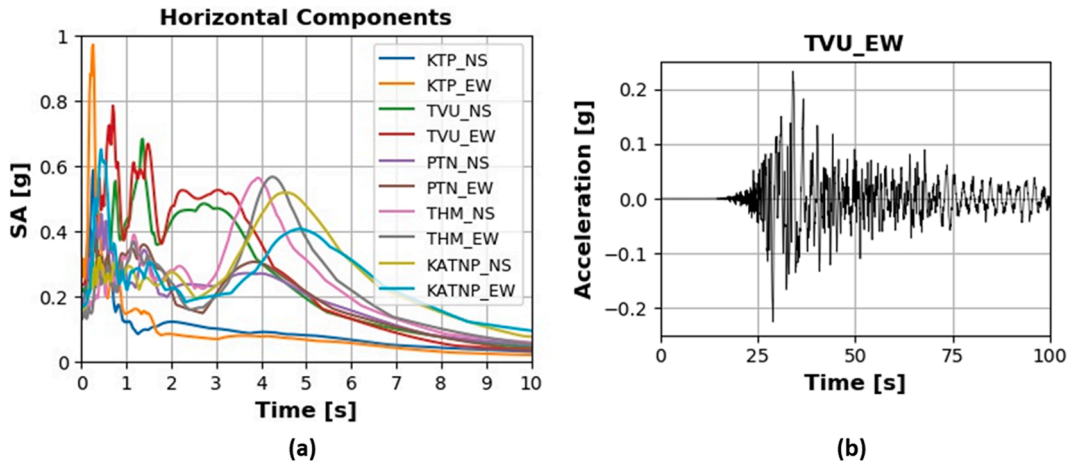
With the aim to further assess the seismic response of the Jaisedewal temple time-history analyses were performed. It is noted that the Jaisedewal temple collapsed during the Gorkha earthquake and thus the seismic records obtained from this earthquake were considered for the time-history analyses.

The epicentre of the earthquake was near the Gorkha region, 80 km from the Kathmandu Valley. Seismic records from the Kathmandu Valley came from the rock site (KTP station) as well as from the sedimentary sites TVU, PTN, THM, and KATNP [65]. The recorded waveforms produced significant spectral accelerations in the range of short periods (up to 1 s) but also for longer periods (i.e. up to 6 s) (Fig. 14a). The largest maximum peak ground acceleration (PGA = 0.25 g) was recorded at the EW component of the KTP





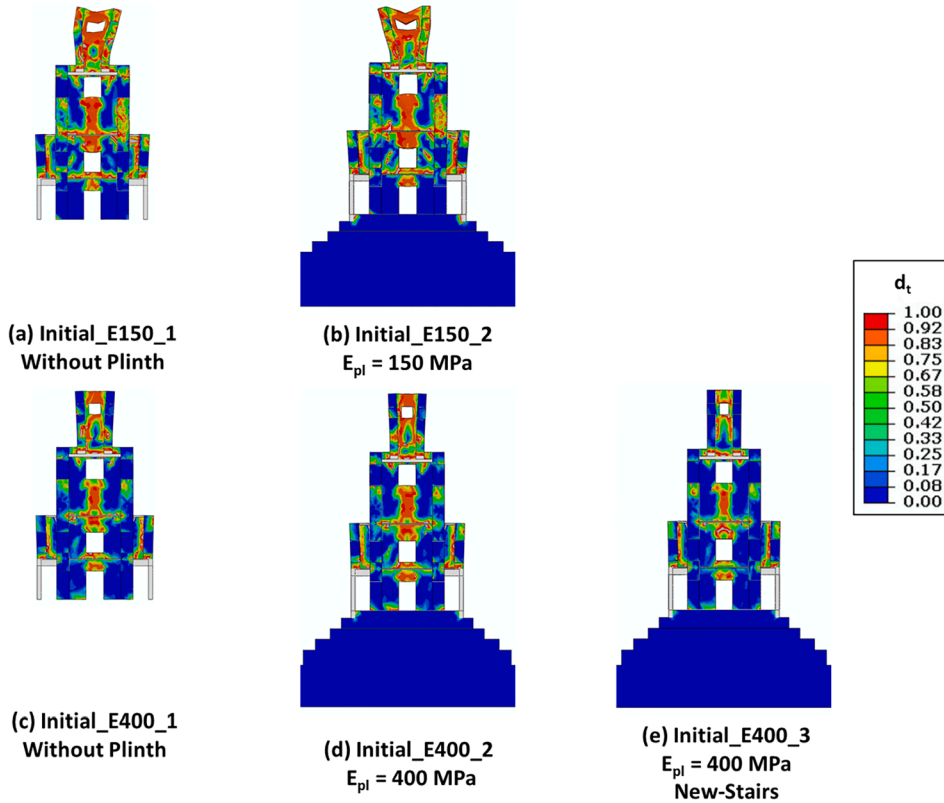
**Fig. 13.** The distribution of tensile damage  $d_t$  for cross-section (top) and perspective view (bottom) of the deformed structure when drift ratio 0.5% is attained during the pushover analysis for (a) the initial (Initial\_E150\_2), and (b) the retrofitted (Retrofitted\_E150\_2) models.



**Fig. 14.** (a) The response spectra corresponding to the horizontal components of the main records of the Gorkha earthquake Nepal (April 25, 2015), and (b) the record used for the time-history analysis.

station. The response spectra of the main records of the Gorkha earthquake are presented in Fig. 14a. The record coming from the EW component of the TVU station [65] was chosen for the time-history analyses performed in this work (Fig. 14b). The distance between the TVU station and the Jaisedewal temple is 3 km. The chosen record has PGA of 0.23 g and more importantly, it produces spectral accelerations of up to approximately 0.8 g at the period of 0.7 s, which coincides with the fundamental period of the Jaisedewal temple. It is highlighted that the long duration of the Gorkha earthquake containing multiple pulses (Fig. 14b) was extremely destructive for the Nepalese monuments. As will be shown below, the Jaisedewal temple reached its capacity in the very first pulses of the excitation, while the next pulses pushed the structure even further making the total collapse inevitable.

During the time-history analysis, the initial model (Initial\_E150\_2) suffers excessive damage from the first seconds of the motion (Fig. 15b). Damage occurs to almost the entire structure and accumulates during the earthquake until collapse. It is underlined that excessive relative deformation is observed between the main masonry core and the peripheral wall. This behaviour points out the lack of proper connection among the structural elements which leads to OOP deformations and possible toppling of the peripheral wall.



**Fig. 15.** The distribution of tensile damage  $d_t$  for cross-section of the deformed structure during the time-history analysis for different configurations of the initial model. (a) and (b) are given for time  $t = 35 \text{ s}$ , while (c), (d) and (e) correspond to the end of the time-history analysis.  $E_{pl}$  refers to the modulus of elasticity assigned to the plinth base. The new-stairs intervention is implemented in (e).

Moreover, an important observation is that the top part of the structure experiences excessive nonlinearity which leads to high drift ratios. In particular, the top of the structure is extremely vulnerable to earthquake vibrations since it does not sit on the main masonry core but is supported by the timber beams. Such failure mode has also been observed in different temples during the Gorkha earthquake (Fig. 7) [54,55]. It is noted that the pushover analysis failed to capture this type of failure which highlights the necessity to perform time-history analysis to accurately evaluate the seismic response of the investigated structural typology. Similar findings were obtained for the initial model with higher material properties (Initial\_E400\_2) and for the models without the plinth base (Initial\_E150\_1 and Initial\_E400\_1); excessive nonlinearity in the temple and thus collapse is inevitable (Fig. 15), while the corresponding drift ratio time-histories are exhibited in Fig. 16. In particular, the maximum drift ratio at the top of the main core for the initial model with low (Initial\_E150\_2) and higher (Initial\_E400\_2) material properties reaches 1.30% and 0.56% respectively. Consequently, the initial model of the temple without any enhancements reveals the high vulnerability of the temple in strong seismic excitations such as the Gorkha earthquake. The numerical results coincide with the actual behaviour of the Jaisedewal temple during the Gorkha earthquake, which led to the collapse of the structure.

As a further step, the initial model with the new-stairs intervention implemented (Initial\_E400\_5) was analysed. This model represents the temple without any interventions at its upper structure while interfering only with the plinth base. In this case, as well, the collapse found to be inevitable (Fig. 15e). Therefore, if interventions are applied only to the plinth base (that is the new-stairs intervention), the seismic response of the temple is not enhanced and further retrofit measures at the upper structure should be considered.

The deformed shape of the retrofitted model with low properties (Retrofitted\_E150\_2) at the end of the time-history analysis is displayed in Fig. 17b. It is noted that even after the retrofit has taken place, the top part of the structure deforms excessively and collapse of that part of the structure seems highly likely. Nevertheless, the response of the rest of the structure has been enhanced and the collapse of the temple is avoided. In particular, the structure presents a box-type behaviour with all the structural elements following the motion of the main core. Therefore, concentration of stresses and OOP deformations are less profound, and the cracks caused by these local failure modes are rather limited. In particular, the OOP deformation of the peripheral wall is zero at the retrofitted structure. Overall, the structure exhibits regions of tensile failures which implies that cracks should be expected on the structure during a similar seismic excitation. The drift ratio of the main core for the Retrofitted\_E150\_2 model is exhibited in Fig. 18b and the maximum recorded drift ratio is 0.47%, a reduction of 64% in comparison to the Initial\_E150\_2 model. The analysis with the Retrofitted\_E150\_2 model terminates before the whole record is applied. This is due to excessive nonlinearities at the top part of the

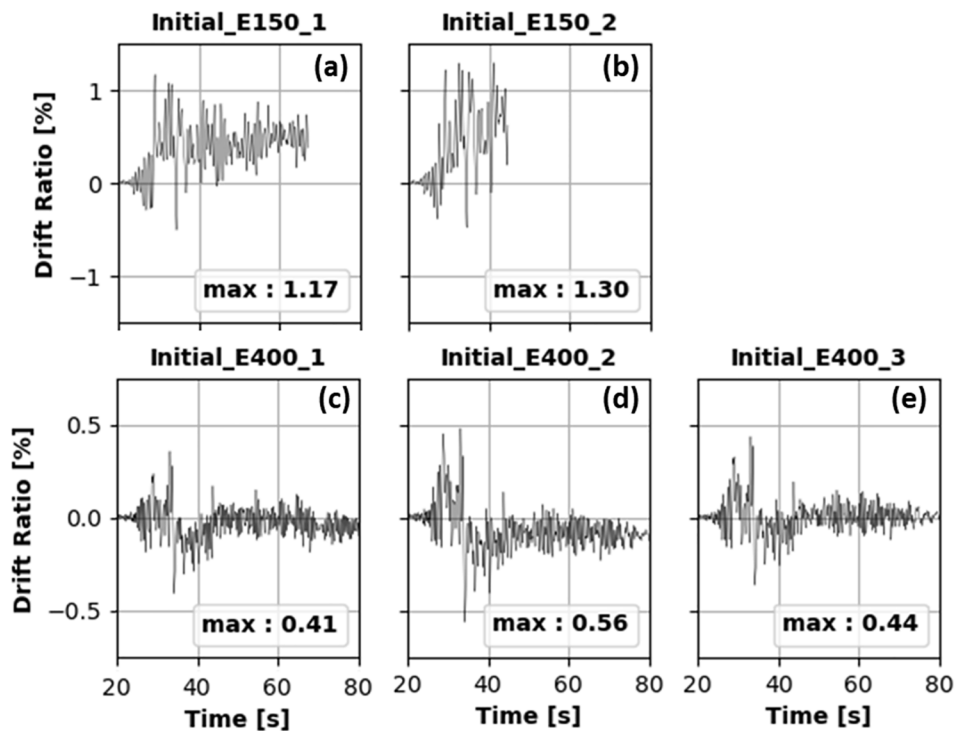


Fig. 16. Time-history of drift ratio at the top of the main core for different configurations of the initial model.

structure. For the retrofitted model with higher properties (Retrofitted\_E400\_2), the collapse of the temple is avoided as well (Fig. 19b) and the recorded drift ratio at the top of the main core reduces by 68% with respect to the Initial\_E400\_2 model. Nevertheless, regions with cracks can still be observed along the structure but are of limited extent.

The deformed shape of the retrofitted models without the plinth base (Retrofitted\_E150\_1 and Retrofitted\_E400\_1) is presented in Fig. 17a and Fig. 19a respectively. It can be concluded that when the plinth base is neglected, the inflicted nonlinearities are less profound and consequently the damage on the temple is underestimated leading to an unrealistic assessment of the monument.

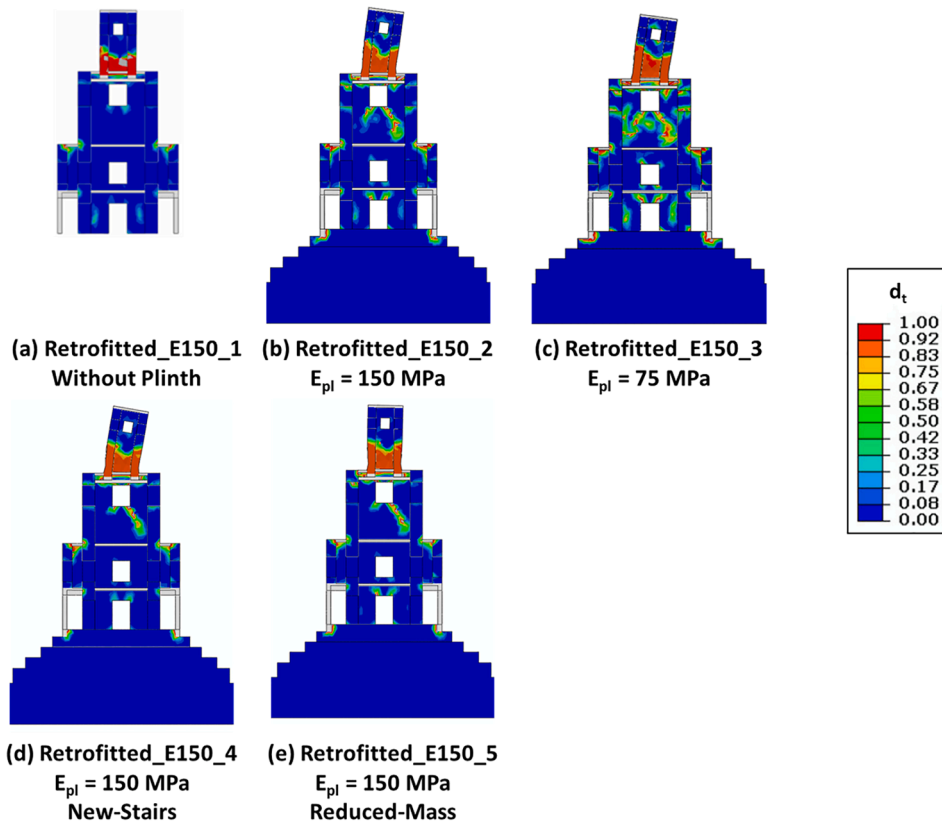
As explained in Section 2.1, usually the plinth base is of lower material properties in comparison to the upper structure. In order to investigate the impact of the properties of the plinth base to the seismic response of the monument different scenarios were examined where the plinth base is of lower properties with respect to the temple. More specifically, for the retrofitted model with low properties, the Retrofitted\_E150\_3 model was developed. In this model the modulus of elasticity for the temple and the plinth base are 150 MPa and 75 MPa respectively. Indeed, the response of the structure changes when the properties of the plinth base deteriorate. Specifically, excessive damage takes place along the structure (Fig. 17c) and the maximum drift ratio is 0.61% (Fig. 18c). It is mentioned that the analysis with the Retrofitted\_E150\_3 model terminates prematurely because of excessive damage at the top part of the structure.

The influence of the properties of the plinth base to the seismic response of the monument was analysed also for the retrofitted model with higher properties. For this purpose, two models were developed, Retrofitted\_E400\_3 and Retrofitted\_E400\_4, with the modulus of elasticity of the plinth base being 150 MPa and 75 MPa respectively. It is reminded that the modulus of elasticity of the temple for these two models is 400 MPa. As a matter of fact, the damage extend to a greater range at the temple as the properties of the plinth base drop (Fig. 19c and d). In addition, the drift ratio increases from 0.18% (Retrofitted\_E150\_2) to 0.30% (Retrofitted\_E400\_3) and 0.43% (Retrofitted\_E400\_4) (Fig. 20c and d).

At this point the effectiveness of the interventions new-stairs and reduced-mass was assessed. In particular, the application of retrofitted plinth base does not seem to have any significant impact at the response of the temple for neither of the retrofitted models with low and higher properties (Retrofitted\_E150\_4 and Retrofitted\_E400\_5) (Fig. 17d and Fig. 19e). On the other hand, the reduced-mass intervention is found to enhance the seismic response of the temple (Retrofitted\_E150\_5 and Retrofitted\_E400\_6) (Fig. 17e and Fig. 19f). In particular, the reduction of the mass of the roofs does not significantly affect the plastification along the structure but the damages at the top part are less profound and collapse might be prevented. When the interventions new-stairs and reduced-mass were integrated in the retrofitted models, the additional reduction of the drift ratio at the top of the main core is only limited, that is 2–8%.

## 7. Discussion

Based on the results from the numerical analyses, it was found that the existence of flexible connections along the Jaisedewal temple needs to be taken into consideration during the modelling in order to retrieve a realistic simulation of the seismic behaviour of the structure. Modelling the top part of the temples, that typically sits on timber elements and is not connected directly to the rest of the



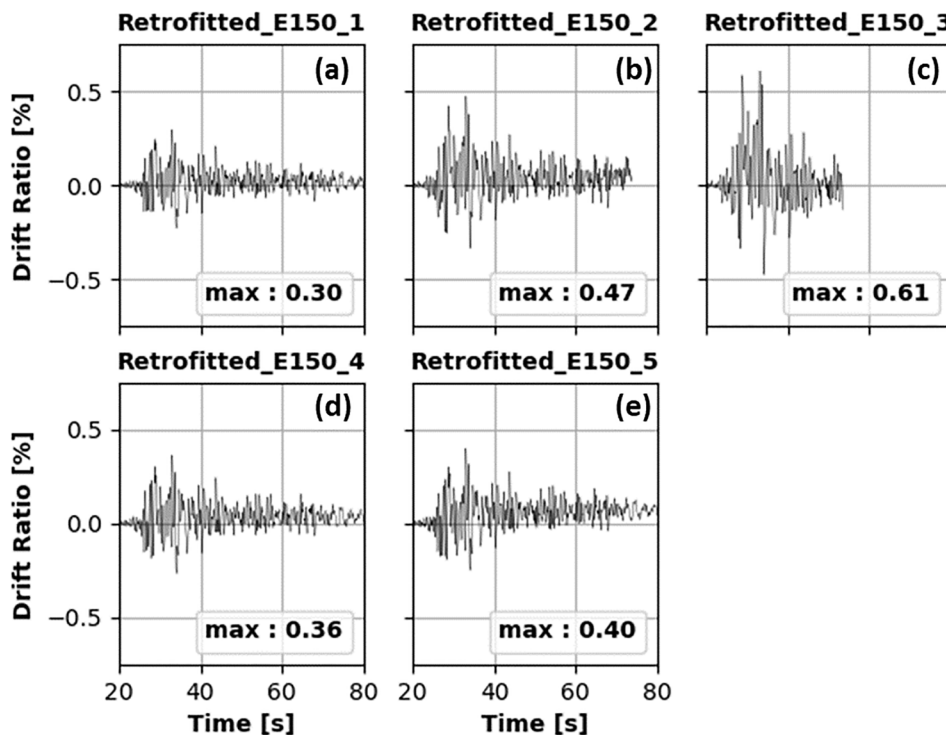
**Fig. 17.** The distribution of tensile damage  $d_t$  for cross-section of the deformed structure during the time-history analysis for different configurations of the retrofitted model with low material properties for the upper structure. (b) and (c) are given for time  $t = 35 \text{ s}$ , while (a), (d) and (e) correspond to the end of the time-history analysis.  $E_{pl}$  refers to the modulus of elasticity assigned to the plinth base. The new-stairs and reduced-mass interventions are implemented in (d) and (e) respectively.

main masonry core, is of paramount importance in order to obtain a precise evaluation of the damage patterns of the structure. The time-history analysis revealed the weak points of the initial (i.e. without any strengthening) structure. The capacity of the temple is exceeded during the very first seconds of the applied excitation. The damage propagates along the whole structure during the next seconds of the record and the collapse is considered inevitable. A retrofitting scheme to improve the seismic capacity of the structure is regarded essential.

The retrofitted model presented an enhanced seismic response of the Jaisedewal temple. Nevertheless, the effectiveness of the interventions depends on the strength of materials of the temple and the plinth base. In particular, for the temple with higher mechanical strength properties, the proposed retrofit reduced the inflicted damage to a minimum level. On the other hand, for the structure with low strength properties, the applied retrofit prevented the collapse of the temple but still damage is expected for a similar seismic excitation. Special care should be given to the top part of the structure that is not directly connected with the main core and was proven to be extremely vulnerable to seismic motion. For the retrofitted models with stiffened-floors, the collapse of the top of the structure is not prevented and further appropriate interventions ought to be examined. This result is in good agreement with in situ inspections of Nepalese monuments after the 2015 Gorkha earthquake. In particular, the top tier roof of the South Taleju Tower, even though it was earlier stiffened in a previous repair and restoration campaign that took place in the period 2013–2015, toppled during the seismic excitation of April 25, 2015 [51] (Fig. 21).

Regarding the response of the plinth base under the seismic excitation, no damage was observed in the core of the plinth in the time-history analyses of the initial and the retrofitted model. Failure occurs only at the connections of the wooden pillars with the base. This behaviour is confirmed from in situ inspections of the Nepalese Pagodas [29,49]. Even in the cases where temples collapsed, no damage had occurred at the plinth base. Nevertheless, although no damage takes place at the plinth itself, the properties of the plinth affect the response of the upper structure as shown from the time-history analyses. In particular, the response of the temple severely deteriorates, and damages extend to a greater range of the temple as the properties of the plinth base drop. When reduced material properties are considered for the plinth base with respect to the temple, the drift ratio at the top of the main core increased by 30% (Retrofitted\_150\_3), 67% (Retrofitted\_E400\_3) and 139% (Retrofitted\_E400\_4). Consequently, special attention is needed to be given to evaluate the properties of the plinth and the investigations should not be limited only to the assessment of the upper structure. Moreover, the incorporation of the plinth base in the numerical model alters the periods of the structure. Thus, when extracting the elastic properties of the temple based on ambient vibrations, the plinth base is suggested to be included in the numerical model,





**Fig. 18.** Time-history of drift ratio at the top of the main core for different configurations of the retrofitted model with low material properties for the upper structure.

otherwise the modes of the structure might not be calculated appropriately.

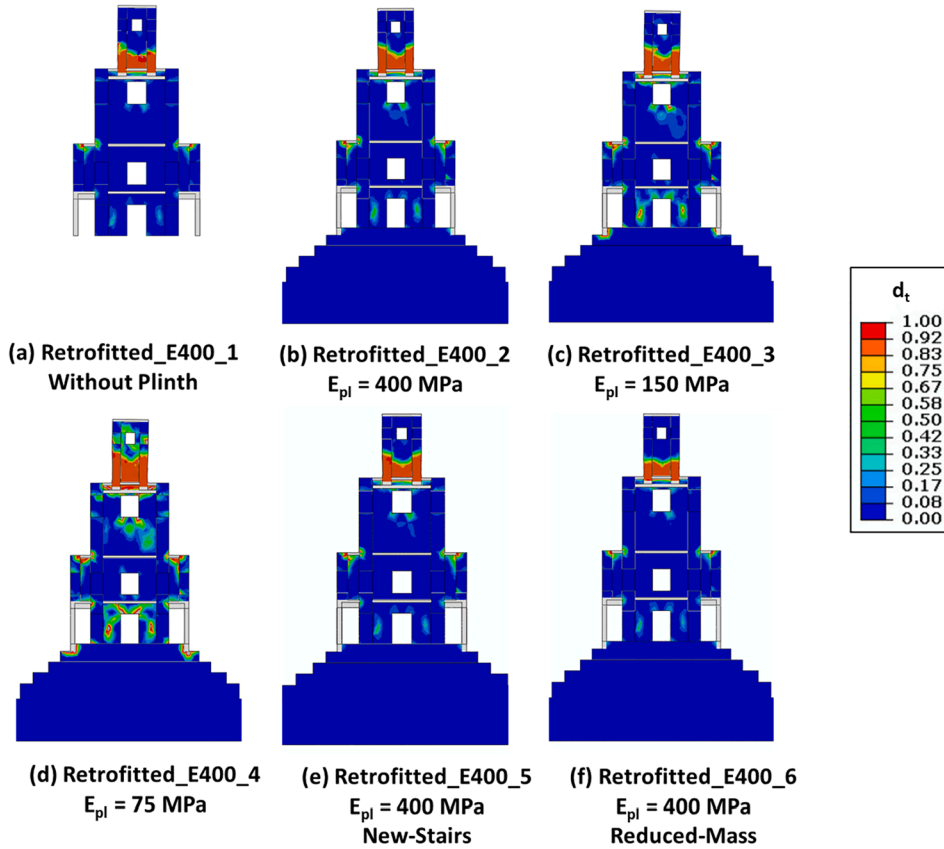
As explained in Section 2.3, the new-stairs intervention is a common practise when rehabilitation schemes are considered for Nepalese Pagodas. Herein, it was numerically shown that its benefits to the structure are insignificant. Taking into consideration that this intervention affects the authentic style of the monument and is visible, it can be concluded that its drawbacks outperform any advantages that it has to offer. Consequently, this intervention is recommended to be dealt with scepticism and perhaps could be avoided.

The mass-reduction of the floors of the temple does not significantly affect the plastification along the structure but the damages at the top part are less profound and the collapse might be prevented. By reducing the mass at the top of the temple, the seismic load transmitted there is reduced and thus the inflicted damage decreases. Thus, it can be inferred that the mass-reduction intervention could be implemented possibly in conjunction with extra measures to further secure the top part of the temple.

The novel structural analysis tools which were used in this study extended traditional methods and highlighted the mechanisms that have allowed the surviving monuments to avoid structural collapse and destruction during strong earthquakes. Also, by better understanding the seismic performance of historic Nepalese pagodas, better decisions on the conservation and rehabilitation techniques could be made. There is much to learn from the forgotten architectural and structural principles developed by the past century builders. The manuscript showcases a clear strategy on the numerical approaches to be used for evaluating the seismic safety of historic monuments in order to prioritise rehabilitation or strengthening and reduce risk to life and livelihoods. It is clear that monuments and historic buildings represent our historic heritage, the witnesses of our history. We have inherited them from the previous generations, and it is our duty to preserve and to transfer them to the future generations. However, the question arises on whether to repair and retrofit them with the aim of preserving the cultural heritage and archaeology or to repair and retrofit them to withstand severe earthquakes and save human lives. This is a multi-dimensional question which requires the assessment of the socio-economic status of a region. From the present study, it became essential to develop conservation strategies and policies based on scientific evidence before we proceed with the restoration of cultural heritage assets in Nepal. Such strategies should involve a multi-disciplinary approach from engineers, archaeologists, economists, social scientists, etc. Furthermore, although this study has mainly focused on Jaisedewal Temple, lessons learned here can be transferred to other temples in the region or even to other tall and slender structures such as historic towers and minarets.

## 8. Conclusions

During the strong 2015 Gorkha earthquake of 7.8  $M_w$  that hit Kathmandu Valley, several historical monuments suffered extensive damage and numerous temples collapsed. For the Nepalese Pagodas, although a significant amount of work has been done to



**Fig. 19.** The distribution of tensile damage  $d_t$  for cross-section of the deformed structure at the end of the time-history analysis for different configurations of the retrofitted model with higher material properties for the upper structure.  $E_{pl}$  refers to the modulus of elasticity assigned to the plinth base. The new-stairs and reduced-mass interventions are implemented in (e) and (f) respectively.

understand their seismic vulnerability, investigations on repair and retrofit options to safeguard these structures from future earthquakes is lacking. In this study, the effectiveness of low-cost, low-interference and reversible repair and strengthening options for the Nepalese Pagodas was evaluated. New materials and construction techniques have been developed since the temples were first built centuries ago. Local engineers and architects working for the restoration of the temples in Nepal were engaged in different aspects of this study to implement a multi-disciplinary assessment and recommend repair and retrofitting solutions which were numerically investigated to assess their effectiveness to withstand large in magnitude earthquakes. These technical recommendations are based on the principles of earthquake engineering and involve reducing the mass of the structure; improve the bonding of the structural elements and connections between structural components; as well as improve the overall resistance of the structure to lateral forces. The three-tiered Jaisedewal temple, typical example of the Nepalese architectural style, was chosen as a case study. A nonlinear three-dimensional finite element model of the Jaisedewal temple was developed in the commercial software ABAQUS based on the macro-modelling approach. Also, modal, pushover and time-history analyses of the temple were performed. The plinth base, that the Nepalese Pagodas usually sit on, has attracted little research and thus its influence on the seismic response of the temple was investigated herein. From the results analysis it was found:

- Pushover analysis cannot simulate the real kinematic nature of a seismic excitation but can provide a rough estimation of some distinctive failure patterns. In particular, pushover analysis was unable to capture the collapse of the top part of the Jaisedewal temple.
- Time-history analysis was deemed essential to accurately reproduce the response of the Jaisedewal temple during a seismic excitation.
- The exposed foundation or otherwise called the plinth base influences the dynamic response of the structure. Even though no damage appears at the core of the plinth base, its strength influences the seismic response of the temple. Thus, the plinth base is highly recommended to be incorporated in the numerical simulation. Also, special attention should be given for the evaluation of the mechanical characteristics of the plinth and the investigations should not be limited only to the assessment of the upper structure.
- Stiffening the connections along the temple was found to enhance the seismic behaviour of the structure and a 64% to 68% reduction of the drift ratio on the top of the main core was achieved.

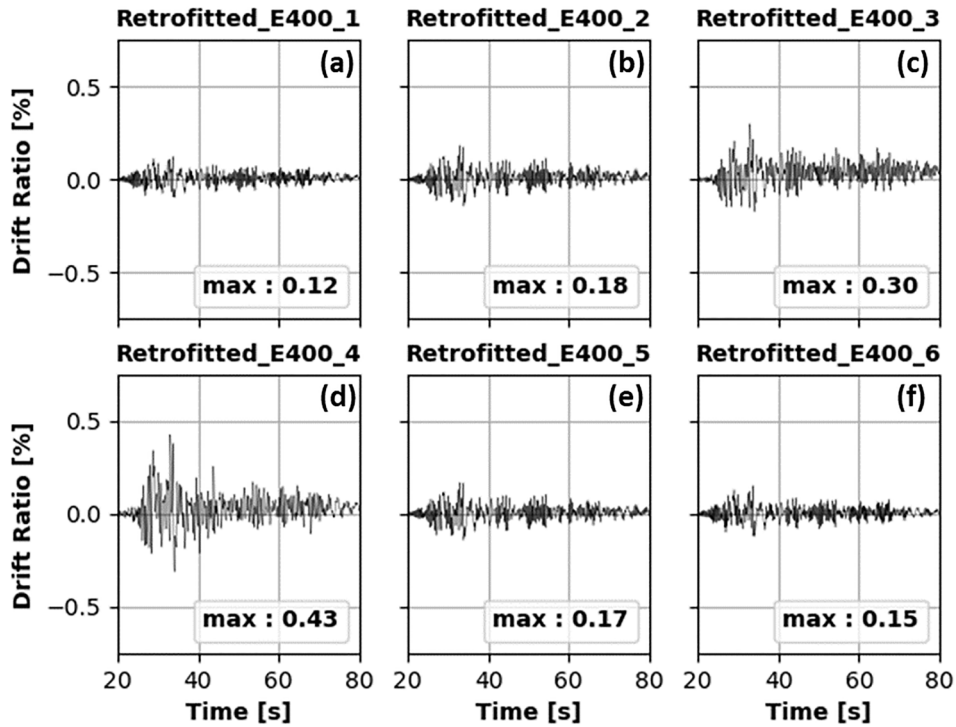


Fig. 20. Time-history of drift ratio at the top of the main core for different configurations of the retrofitted model with higher material properties for the upper structure.



Fig. 21. The top tier roof of the South Taleju Tower toppled during the 2015 Gorkha earthquake [51].

- Currently, for the reconstruction of the temples, the perimetry of the exposed foundations is rebuild with new bricks bonded together with high strength mortar. Herein, it was numerically shown that performing such intervention does not improve the seismic vulnerability of the structure; the drift ratio decreased only by 2% to 8%. Now, taking into consideration that this

intervention affects the authentic style of the monument and is visible, it can be concluded that its drawbacks outperform any advantages that it has to offer. Consequently, this intervention is recommended to be avoided.

- While reducing the masses at the roof of the Jaisedewal temple only slightly enhanced the overall response of the monument, i.e. the drift ratio at the top of the main core was reduced by 5%, this intervention decreased the inflicted damage at the top part of the structure. Thus, this intervention should be implemented in conjunction with other measures to further strengthen the top part of the temple.

Overall, the Nepalese Pagodas are historical monuments characterised by peculiarity and architectural significance. Therefore, to develop a holistic scheme for their seismic intervention, further studies need to be undertaken. These studies should mainly focus on understanding the soil characteristics of the area, the soil-structure interaction phenomena during strong earthquakes, performing destructive and non-destructive tests to understand the mechanical response of the timber beams, masonry units and mortar joints etc. Moreover, educating and preparing the local engineers, planners and policy makers is essential so that they take control of the reconstruction processes and heritage protection.

## Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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